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CONCRETE
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Volume LIV, No. 10. LONDON, OCTOBER, 1959.

EDITORIAL NOTES

An Accelerated Test for Cement.

THE use of accelerated tests is beset with pitfalls and dangers. This is, of course, common knowledge, but it is surprising how the desire to market a product quickly can tempt firms to accept such tests as a true indication of the performance of a product over a longer period or in conditions different from those used in the tests. Accelerated tests of the resistance of materials to the weather often have little relation to the conditions to which they will be subjected when they are in actual use. For example, accelerated tests that are supposed to indicate the resistance of a material when it is exposed to low temperatures in the winter and high temperatures in the summer do nothing of the sort. Rapid alternations of freezing and thawing during a short period in a laboratory apparatus will prove the durability of a material in these conditions, but they give no indication of what will happen when it is exposed to low temperatures for weeks on end or when it is subjected to continuously high temperatures due to climate or industrial processes. The results of accelerated tests can be as misleading and dangerous as extrapolating a curve, and can be the cause of much loss and disappointment.

In the case of concrete it is common to specify sufficient strength at twenty-eight days to ensure that a structure will carry the working load for which it is designed, because the setting and hardening properties of ordinary Portland cement are such that it will not always achieve the required strength at an earlier age and also because it is only rarely that a structure is subjected to its working load at an earlier age. Cement and concrete continue to gain strength at later ages, but a knowledge of the strength it will have at twenty-eight days generally has to be sufficient if undue delay in construction is to be avoided. It is desirable to know as quickly as possible the most economical mixture that will produce the required strength, and many tests have established a ratio that is reasonably reliable between the properties at seven days and at twenty-eight days. If the work is urgent, if the sources of the cement and aggregate may vary during the period of construction, or if a slower-hardening cement may be used, it is desirable to have a reliable test that will give an idea in less than seven days of the strength of cement or concrete at twenty-eight days. However, when ordinary Portland cement and ordinary methods of storing test specimens are used we are confronted with the fact that at very early ages the cement is so

weak that the specimen would be destroyed before the load applied to it was great enough to give useful results.

In the construction of dams in Switzerland it was desired to avoid the cost of building the silos that would be necessary to store cement on the site if it were not used until tests were made of specimens cast seven days previously. The problem was investigated by the Swiss Federal Laboratory for Testing Materials and Research and a method was devised by which specimens tested thirty-six hours after they were cast gave a remarkably reliable indication of the strength that could be expected at twenty-eight days. It was found that the compressive strengths obtained in the accelerated tests and in the standard tests were in fact for all practical purposes identical. Tests were made on more than seven hundred specimens made with eleven different brands of Portland cement at the Federal Laboratory and also on 546 specimens at seven other laboratories in Switzerland. All the specimens in these tests measured 4 cm. by 4 cm. by 16 cm. and comprised one part of cement to three parts of sand in accordance with the Swiss standard for testing mortar. A total of 129 specimens tested at three different laboratories had an average strength of 7620 lb. per square inch in the accelerated tests and 7634 lb. per square inch in the case of the check specimens tested at twenty-eight days. Except in a very few instances in which the specimens failed in the test the differences between the highest and lowest strengths obtained were about the same in both sets of tests, and were within the range commonly met with in laboratory tests.

The accelerated test is as follows. During the first twenty-four hours after casting the specimens are stored in air at a temperature of 65 deg. Fahr. and a relative humidity of at least 95 per cent. They are then put into an autoclave and covered with water and the bottom heating element is switched on until steam emerges from it in about one hour. The steam outlet is then closed and the side heating elements are switched on until in about one hour the pressure is twelve atmospheres and this is maintained for three hours, after which the heat is switched off, the steam outlet is opened, and the specimens are allowed to cool to 200 deg. Fahr. in a period of three hours. The specimens are then removed from the autoclave, placed in hot water, allowed to cool to the temperature of the atmosphere during a period of one hour, and are then ready for crushing tests.

It is pointed out that while the accelerated test has been proved to be an accurate guide to the compressive strength at twenty-eight days of the cements tested, the same result is not obtained in the case of bending tests made in accordance with the Swiss Standard. The strengths in bending increased less than the compressive strength in the accelerated tests.

Further study of these tests showed that when the mortar was heated in the autoclave some of the chemical reactions that took place were different from those which occurred when the rate of hardening was slower. The compressive strengths were, however, the same twenty-eight days after the specimens were cast, and specimens that were tested a year later also had the same strengths. In the case of Portland cement it is fortunate yet fortuitous that although the chemical process of hardening is different it does not affect the later development of strength, so that the accelerated test is a reliable guide to the strength of the material at later ages. Full details of the results of these tests are given in Report No. 188 of the Swiss Federal Laboratory for Testing Materials, Zurich.

Bending Moments on Inter-connected Beams.

By L. S. MÜLLER.

An approximate method for the calculation of the maximum bending moments on inter-connected beams freely supported at their outer ends by unyielding supports has been given by Professor Saliger.[1] The torsional restraint of one beam on another is neglected ; the deflections at the centres of imaginary middle beams, of unit widths in the directions x and y intersecting at M in *Fig.* 1, are assumed to be the same. If the partial loads in directions x and y are w_x and w_y, and the total load is

$$w = w_x + w_y \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

then, with the notation of *Fig.* 1,

$$\frac{5w_xL_x^4}{384\,EI_x} = \frac{5w_yL_y^4}{384\,EI_y} \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Assuming that $I_x = I_y$, then, from (1) and (2),

$$w_y = \frac{L_x^4}{L_x^4 + L_y^4}w = \alpha w, \quad \text{and} \quad w_x = \frac{L_y^4}{L_x^4 + L_y^4}w = (1 - \alpha)w.$$

The load on the middle beam is the load per unit area multiplied by the spacing of the beams, that is $w_x\frac{L_y}{m+1}$ in direction x, and $w_y\frac{L_x}{n+1}$ in direction y.

Each of the other beams parallel to the directions x and y resists a bending moment in the ratio of its central deflection to that of the imaginary parallel

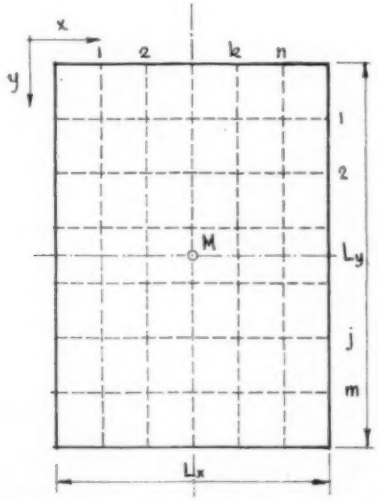


Fig. 1.—Simply-supported Panel of Inter-connected Beams.

middle beam. The ratio of the deflection f_k of beam k to the deflection f_M of the imaginary middle beam, both spanning in direction y, is

$$\frac{f_k}{f_M} = \frac{16}{5}\left[\frac{k}{n+1} - 2\left(\frac{k}{n+1}\right)^3 + \left(\frac{k}{n+1}\right)^4\right] \quad . \quad . \quad . \quad (3)$$

Values of k from 1 to n, and in direction x from 1 to m, have been calculated from equation (3) and are given in *Table* I.

If the maximum bending moment on each beam is assumed to be $\frac{wL^2}{8}$ and not $\frac{wL^2}{10}$ as originally proposed by Professor Saliger, then, according to Dr. Genttner,[2] the maximum bending moments on the other beams agree very well with those computed by Dr. Genttner's method, which takes torsional restraint into account. Away from the points of maximum bending moment, Professor

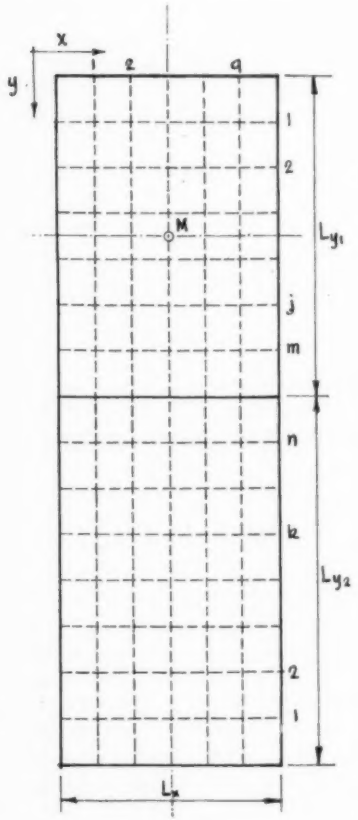


Fig. 2.

TABLE I.—RATIOS OF DEFLECTIONS OF SIMPLY-SUPPORTED PANELS.

n	k	f_k/f_M	n	k	f_k/f_M
2	1;2	0·869	9	1;9	0·314
3	1;3	0·713		2;8	0·594
4	1;4	0·594		3;7	0·813
	2;3	0·952		4;6	0·952
5	1;5	0·506	10	1;10	0·286
	2;4	0·869		2;9	0·547
6	1;6	0·440		3;8	0·761
	2;5	0·787		4;7	0·912
	3;4	0·976		5;6	0·990
7	1;7	0·388	11	1;11	0·263
	2;6	0·713		2;10	0·506
	3;5	0·926		3;9	0·713
8	1;8	0·347		4;8	0·869
	2;7	0·649		5;7	0·966
	3;6	0·869			
	4;5	0·985			

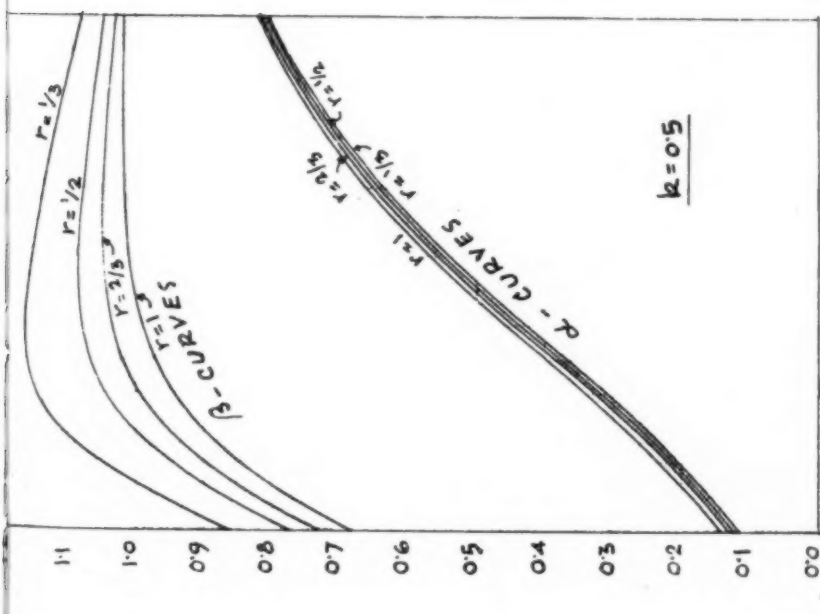

k=0·5
α - CURVES
r=1
r=2/3
r=1/2
r=1/3
β - CURVES
r=1/3
r=1/2
r=2/3
r=1
λ1=0·5
0·75
1·00
1·25
0·0
0·1
0·2
0·3
0·4
0·5
0·6
0·7
0·8
0·9
1·0
1·1

Fig. 3.

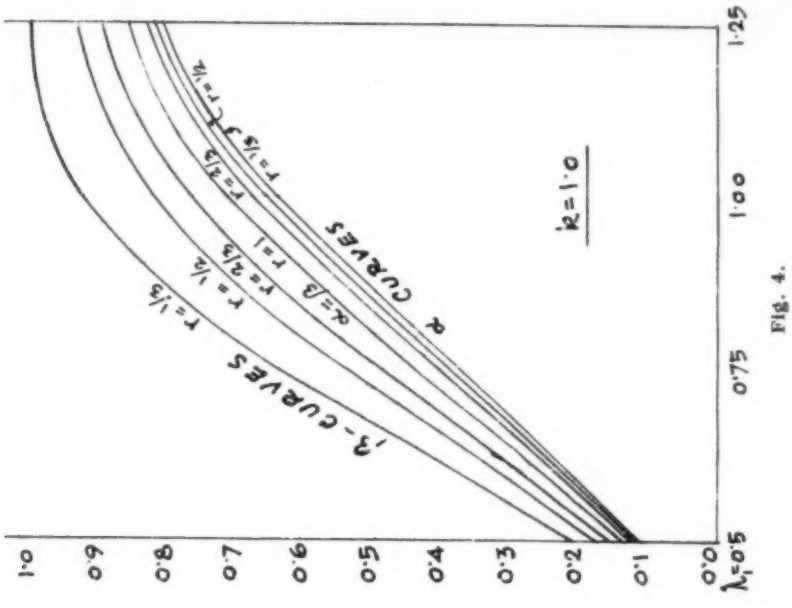

k=1·0
α CURVES
r=1/3
r=1/2
r=2/3
r=1
r=3/2
β - CURVES
r=1/3
r=2/3
λ1=0·5
0·75
1·00
1·25
0·0
0·1
0·2
0·3
0·4
0·5
0·6
0·7
0·8
0·9
1·0

Fig. 4.

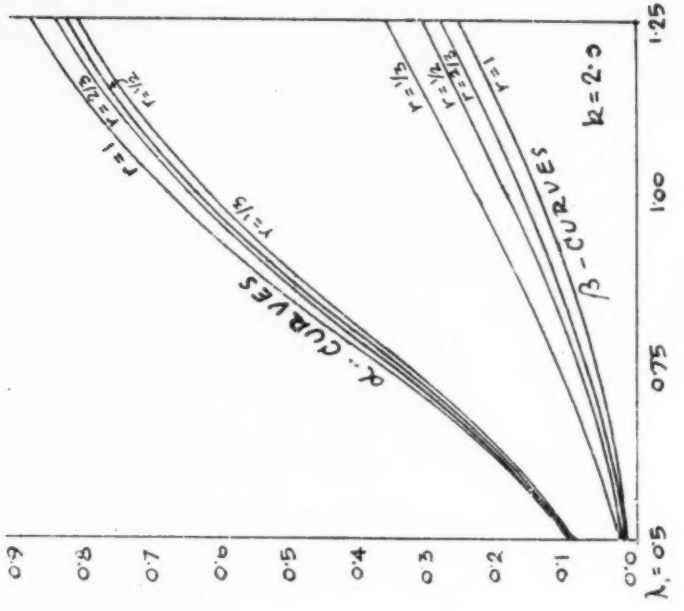

k=2·0
β - CURVES
α - CURVES
r=1
r=2/3
r=1/2
r=1/3
λ=0·5
0·75
1·00
1·25
0·0
0·1
0·2
0·3
0·4
0·5
0·6
0·7
0·8
0·9

Fig. 6.

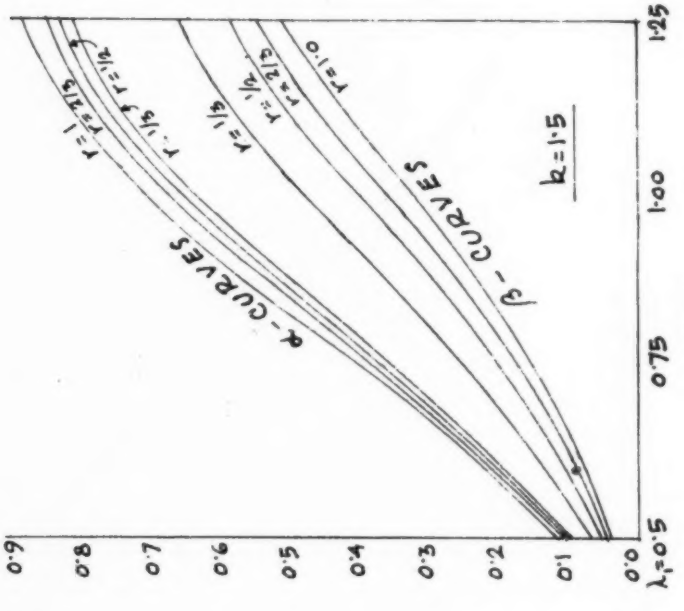

k=1·5
β - CURVES
α - CURVES
r=1·0
r=1/2
r=1/3
r=2/3
r=1
λ=0·5
0·75
1·00
1·25
0·0
0·1
0·2
0·3
0·4
0·5
0·6
0·7
0·8
0·9

Fig. 5.

Saliger's method gives somewhat lower values than those calculated by Dr. Genttner's method because the bending-moment diagrams for the longer beams are not simple parabolas.

Professor Saliger's method can also be applied to two adjoining panels of beams, continuous in one direction and freely supported on the other sides (*Fig.* 2), if the same conditions as before are assumed for the supports, including the internal support in the direction of continuity.

If the total loads on panels 1 and 2 are w_1 and w_2, the partial loads are

$$w_{1y} = \alpha w_1, \quad w_{1x} = (1 - \alpha)w_1. \quad w_{2y} = \beta w_2, \quad w_{2x} = (1 - \beta)w_2.$$

Due to these loads, the bending moment at the support of the imaginary middle beam of unit width in direction y is

$$M_{sy} = -\frac{w_{1y}L_{1y}^3 + w_{2y}L_{2y}^3}{8(L_{1y} + L_{2y})},$$

and for equal deflections at the mid-point of panel 1,

$$\frac{5w_{1y}L_{1y}^4}{384\,EI_y} - \frac{w_{1y}L_{1y}^3 + w_{2y}L_{2y}^3}{8(L_{1y} + L_{2y})} \cdot \frac{L_{1y}^2}{16EI_y} = \frac{5}{384} \cdot \frac{w_{1x}L_x^4}{EI_x} \quad . \quad . \quad (4)$$

from which

$$\frac{\alpha w_1 L_{1y}^4}{384EI_y}\left\{5 - \frac{3\left[1 + \frac{\beta}{\alpha} \cdot \frac{w_2}{w_1}\left(\frac{L_{2y}}{L_{1y}}\right)^3\right]}{1 + \frac{L_{2y}}{L_{1y}}}\right\} = \frac{5}{384} \cdot \frac{(1 - \alpha)w_1 L_x^4}{EI_x}.$$

Writing $\frac{w_2}{w_1} = r$, $\frac{L_{2y}}{L_{1y}} = k$, and $I_x = I_y$,

$$\alpha L_{1y}^4\left[5 - \frac{3\left(1 + \frac{\beta}{\alpha} r k^3\right)}{1 + k}\right] = 5(1 - \alpha)L_x^4 \quad . \quad . \quad . \quad (5)$$

Similarly for panel 2,

$$\beta L_{2y}^4\left[5 - \frac{3\left(1 + \frac{\alpha}{\beta} \cdot \frac{1}{r} \cdot \frac{1}{k^3}\right)}{1 + \frac{1}{k}}\right] = 5(1 - \beta)L_x^4 \quad . \quad . \quad . \quad (6)$$

From equations (5) and (6),

$$5 - \frac{3\left(1 + \frac{\beta}{\alpha} r k^3\right)}{1 + k} = 5\left(\frac{1 - \alpha}{\alpha}\right)\left(\frac{L_x}{L_{1y}}\right)^4 \cdot \quad 5 - \frac{3\left(1 + \frac{\alpha}{\beta r k^3}\right)}{1 + \frac{1}{k}} = 5\left(\frac{1 - \beta}{\beta}\right)\left(\frac{L_x}{L_{2y}}\right)^4 \quad (7)$$

With $\frac{L_x}{L_{1y}} = \lambda_1$ and $\frac{L_x}{L_{2y}} = \lambda_2 = \frac{\lambda_1}{k}$, equation (7) can be solved for α and β.

$$[2 + 5k + 5(1 + k)\lambda_1^4]\alpha - 3k^3 r\beta = 5(1 + k)\lambda_1^4 \quad . \quad . \quad (8)$$

$$\left[2 + \frac{5}{k} + \frac{5(1 + k)}{k}\lambda_2^4\right]\beta - \frac{3}{k^3 r}\alpha = \frac{5(1 + k)}{k}\lambda_2^4. \quad . \quad . \quad (9)$$

From equation (8),

$$\beta = \frac{[2 + 5k + 5\lambda_1{}^4(1 + k)]\alpha - 5(1 + k)\lambda_1{}^4}{3rk^3} \quad . \quad . \quad . \quad (10)$$

Inserting this value in equation (9),

$$\alpha = \frac{(k + 1)\left(2 + \frac{5\lambda_1{}^4}{k^4}\right) + \frac{3(k + r)}{k}}{(k + 1)\left(\frac{2}{\lambda_1{}^4} + \frac{5\lambda_1{}^4}{k^4}\right) + \frac{1}{k^3}\left[\frac{2(k^5 + 1)}{k} + 5(k^3 + 1)\right]} \quad . \quad . \quad (11)$$

Equations (10) and (11) have been used in constructing *Figs.* 3 to 6. For the calculation of related values of α and β the following values are assumed: For k: 0·5, 1·0, 1·5, and 2·0. For r: $\frac{1}{3}$, $\frac{1}{2}$, $\frac{2}{3}$, and 1. For λ_1: from 0·5 to 1·25.

If the load on the second panel is much less than that on the first and the second span in the direction of continuity is relatively small, k and r are small

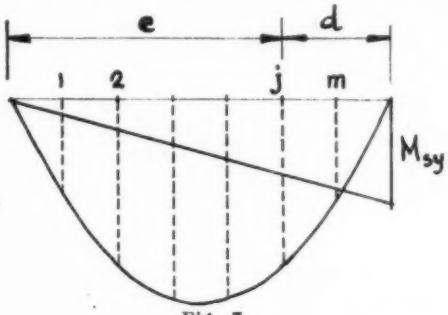


Fig. 7.

and λ_1 is large, and β may be greater than unity. This shows that the load on panel 1 is tending to lift panel 2, so that the beams in the direction x in panel 2 will be subjected to an upward load since $(1 - \beta)$ is negative; alternatively the beams in the direction y in panel 2 will receive an additional load due to the restraining effect of the beams in the direction x.

Having determined α, β, $1 - \alpha$, and $1 - \beta$, the ratios of the deflection of each beam at its intersection with the middle beam to the deflection at the mid-point of the middle beam must be found. For the beams in the direction x, with free supports, the ratios may be taken from *Table* I, substituting q for n. The ratios in the direction of continuity are obtained as follows. With the notation of *Fig.* 7,

$$d = \frac{m + 1 - j}{m + 1}.L_{1y}, \text{ and } e = \frac{j}{m + 1}.L_{1y}.$$

The moment at j is $M_j = \frac{\alpha w_1 de}{2} - \frac{e}{L_{1y}}.M_{sy}$.

The reaction at the left-hand support when the beam is loaded by the moment-area diagram is

$$R_1 = \frac{\alpha w_1 L_{1y}{}^3}{24} - \frac{M_{sy}L_{1y}}{6}, \text{ and } f_j = R_1 e - \frac{\alpha w_1 e^3}{24}(2L_{1y} - e) + \frac{e^3 M_{sy}}{6L_{1y}}.$$

TABLE II.

m \ J	1	2	3	4	5	6	7	8	9	10	11
2	a = 0·5926 b = 7·3330 c = 4·0000	1·1852 7·6670 2·5000									
3	a = 0·250 b = 14·250 c = 7·500	0·500 10·000 6·000	0·750 4·750 3·500								
4	a = 0·128 b = 23·200 c = 12·000	0·256 18·600 10·500	0·384 12·400 8·000	0·512 5·800 4·500							
5	a = 0·0741 b = 34·1667 c = 17·5000	0·1481 29·3333 16·0000	0·2222 22·5000 13·5000	0·2963 14·6667 10·0000	0·3704 6·8333 5·5000						
6	a = 0·0466 b = 47·1429 c = 24·0000	0·0933 42·1429 22·5000	0·1399 34·8572 20·0000	0·1866 26·1429 16·5000	0·2332 16·8572 12·0000	0·2799 7·8571 6·5000					
7	a = 0·0313 b = 62·1250 c = 31·5000	0·0625 57·0000 30·0000	0·0938 49·3750 27·5000	0·1250 40·0000 24·0000	0·1563 29·6250 19·5000	0·1875 19·0000 14·0000	0·2188 8·5750 7·5000				
8	a = 0·0220 b = 79·1111 c = 40·0000	0·0439 73·8889 38·5000	0·0658 66·0000 36·0000	0·0878 56·1111 32·5000	0·1097 44·8889 28·0000	0·1317 33·0000 22·5000	0·1536 21·1111 16·0000	0·1756 9·8889 8·5000			
9	a = 0·016 b = 98·100 c = 49·500	0·032 92·800 48·000	0·048 84·700 45·500	0·064 74·400 42·000	0·080 62·500 37·500	0·096 49·600 32·000	0·112 36·300 25·500	0·128 23·200 18·000	0·1440 10·9000 9·5000		
10	a = 0·0202 b = 119·0300 c = 60·0000	0·0404 115·7730 58·5000	0·0606 105·4350 56·0000	0·04808 94·8180 52·5000	0·06011 82·3636 48·0000	0·07213 68·6360 42·5000	0·0845 54·1818 34·0000	0·09617 39·5455 28·5000	0·10819 25·2727 20·0000	0·12021 11·9090 10·5000	
11	a = 0·00926 b = 142·08330 c = 71·50000	0·01852 136·66670 70·00000	0·02778 128·25000 67·50000	0·03704 117·33330 64·00000	0·0463 104·4167 59·5000	0·05556 90·00000 54·00000	0·06481 74·58330 47·50000	0·07407 58·66670 40·00000	0·08333 42·75000 31·50000	0·09259 27·33333 22·00000	0·10185 12·91670 11·50000

As $M_{sy} = \alpha w_1 L_{1y}^2 \left[\frac{1 + \frac{\beta}{\alpha} rk^3}{8(1+k)}\right]$,

$$f_j = \frac{\alpha w_1 L_{1y}^4 j}{24(m+1)^3}\left\{\frac{(m+1)^3 - j^2(2m-j+2)}{m+1} - \frac{\left[1 + \frac{\beta}{\alpha} rk^3\right][(m+1)^2 - j^2]}{2(1+k)}\right\}\frac{1}{EI_y^b},$$

and $f_M = \frac{\alpha w_1 L_{1y}^4}{128}\left[\frac{5}{3} - \frac{1 + \frac{\beta}{\alpha} rk^3}{1+k}\right]\frac{1}{EI_y^b}$.

$$\frac{f_j}{f_M} = \frac{16j}{(m+1)^3} \cdot \frac{\frac{(m+1)^3 - j^2(2m-j+2)}{m+1} - \frac{\left[1 + \frac{\beta}{\alpha} rk^3\right][(m+1)^2 - j^2]}{2(1+k)}}{5 - \frac{3\left(1 + \frac{\beta}{\alpha} rk^3\right)}{1+k}} \quad . \quad (12)$$

Writing

$$\frac{16j}{(m+1)^3} = a, \quad \frac{(m+1)^3 - j^2(2m-j+2)}{(m+1)} = b, \text{ and } \frac{(m+1)^2 - j^2}{2} = c,$$

these coefficients are independent of the loads and are functions of the number of cross beams, their ratio in both panels, and the proportions of the panels.

Equation (12) becomes $\frac{f_j}{f_M} = \frac{a(b - c\zeta)}{5 - 3\zeta}$ . . . . . (13)

in which $\zeta = \frac{1 + \frac{\beta}{\alpha} rk^3}{1+k} = \phi(\lambda_1, k, r)$.

The values of a, b, and c for $j = 1$ to $j = m$, and from $m = 2$ to $m = 11$, are given in *Table* II.

Examples.

For comparison the same size of panel is used in the following, first as a single panel freely supported on all four sides, and secondly as part of two panels continuous in the direction of the longer side.

EXAMPLE I.—For the single panel freely supported on all four sides (*Fig.* 1), $L_x = 25$ ft. 6 in., $L_y = 34$ ft., $n = 5$, $m = 7$; that is the centres of the beams in both directions are 4 ft. 3 in. apart. Assuming the dimensions shown on *Fig.* 8 the weight g of the panel is 110 lb. per square foot; the live load p is 200 lb. per square foot, and the total load w is 310 lb. per square foot. Then

$$w_x = \frac{34^4}{25{\cdot}5^4 + 34^4} \times 310 = 0{\cdot}76 \times 310 = 236 \text{ lb. per sq. ft., and}$$

$$w_y = \frac{25{\cdot}5^4}{25{\cdot}5^4 + 34^4} \times 310 = 0{\cdot}24 \times 310 = 74 \text{ lb. per sq. ft.}$$

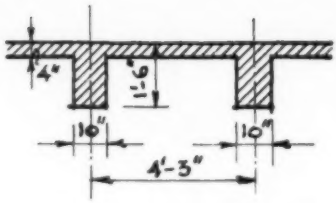


Fig. 8.—Section through Part of a Panel.

The loads on the beams in the direction y are $w_{3y} = 4{\cdot}25 \times 74 = 315$ lb. per linear foot on the middle beam, and, from *Table* I, for $n = 5$,

$$w_{2y} = w_{4y} = 0{\cdot}869 \times 315 = 274 \text{ lb. per foot and}$$
$$w_{1y} = w_{5y} = 0{\cdot}56 \times 315 = 176{\cdot}5 \text{ lb. per foot}$$

on the other beams. In the direction x the loads are $w_{4x} = 4{\cdot}25 \times 236 = 1000$ lb. per linear foot on the middle beam, and, from *Table* I, for $n = 7$,

$$w_{3x} = w_{5x} = 0{\cdot}926 \times 1000 = 926 \text{ lb. per ft.,}$$
$$w_{2x} = w_{6x} = 0{\cdot}713 \times 1000 = 713 \text{ lb. per ft., and}$$
$$w_{1x} = w_{7x} = 0{\cdot}388 \times 1000 = 388 \text{ lb. per ft.}$$

Therefore, the bending moments in the direction y are

$$M_{3y(\max.)} = \frac{315 \times 34^2}{8} = 45{,}500 \text{ ft.-lb.;}$$

$$M_{2y(\max.)} = M_{4y(\max.)} = \frac{274 \times 34^2}{8} = 39{,}600 \text{ ft.-lb.;}$$

$$M_{1y(\max.)} = M_{5y(\max.)} = \frac{176{\cdot}5 \times 34^2}{8} = 25{,}500 \text{ ft.-lb.}$$

The bending moments in the direction x are

$$M_{4x(\max.)} = \frac{1000 \times 25{\cdot}5^2}{8} = 81{,}500 \text{ ft.-lb.;}$$

$$M_{3x(\max.)} = M_{5x(\max.)} = \frac{926 \times 25{\cdot}5^2}{8} = 75{,}300 \text{ ft.-lb.;}$$

$$M_{2x(\max.)} = M_{6x(\max.)} = \frac{713 \times 25{\cdot}5^2}{8} = 58{,}100 \text{ ft.-lb.;}$$

$$M_{1x(\max.)} = M_{7x(\max.)} = \frac{388 \times 25{\cdot}5^2}{8} = 31{,}600 \text{ ft.-lb.}$$

EXAMPLE II.—For two panels continuous in the longer direction (*Fig.* 2), $L_x = 25$ ft. 6 in., $q = 5$, $L_{y1} = L_{y2} = 34$ ft., $m = n = 7$. The spacing of the beams, and the loads, are as given in Example I.

For the beams in the direction x, spanning 25 ft. 6 in., the maximum bending moment occurs when one panel only is loaded.

$$r = \frac{g}{w} = \frac{110}{310} = 0{\cdot}355, \quad \lambda_1 = \frac{25{\cdot}5}{34{\cdot}0} = 0{\cdot}75, \text{ and } k = 1.$$

From *Fig.* 4, $\alpha = 0{\cdot}377$, $\beta = 0{\cdot}630$, and $1 - \alpha = 0{\cdot}623$.

For the loaded panel, $w_y = 0{\cdot}377 \times 310 = 116{\cdot}5$ lb. per square foot,

$w_x = 0{\cdot}623 \times 310 = 193{\cdot}5$ lb. per square foot, and

$$\zeta = \frac{1 + \frac{0{\cdot}63}{0{\cdot}377} \times 0{\cdot}355}{1 + 1} = 0{\cdot}805.$$

From equation (13) and *Table* II:

$$\frac{f_1}{f_M} = \frac{0{\cdot}0313}{5 - 3 \times 0{\cdot}805}(62{\cdot}125 - 31{\cdot}5 \times 0{\cdot}805) = 0{\cdot}444;$$

$$w_{1x} = 4{\cdot}25 \times 0{\cdot}444 \times 193{\cdot}5 = 365 \text{ lb. per ft.}$$

$$\frac{f_2}{f_M} = \frac{0{\cdot}0625}{2{\cdot}585}(57{\cdot}0 - 30 \times 0{\cdot}805) = 0{\cdot}795;$$

$$w_{2x} = 4{\cdot}25 \times 0{\cdot}795 \times 193{\cdot}5 = 655 \text{ lb. per ft.}$$

$$\frac{f_3}{f_M} = \frac{0{\cdot}0978}{2{\cdot}585}(49{\cdot}375 - 27{\cdot}5 \times 0{\cdot}805) = 1{\cdot}03;$$

$$w_{3x} = 4{\cdot}25 \times 1{\cdot}03 \times 193{\cdot}5 = 850 \text{ lb. per ft.}$$

$$\frac{f_4}{f_M} = \frac{0{\cdot}125}{2{\cdot}585}(40 - 24 \times 0{\cdot}805) = 1{\cdot}00;$$

$$w_{4x} = 4{\cdot}25 \times 1{\cdot}00 \times 193{\cdot}5 = 820 \text{ lb. per ft.}$$

$$\frac{f_5}{f_M} = \frac{0{\cdot}1563}{2{\cdot}585}(29{\cdot}625 - 19{\cdot}5 \times 0{\cdot}805) = 0{\cdot}842;$$

$$w_{5x} = 4{\cdot}25 \times 0{\cdot}842 \times 193{\cdot}5 = 695 \text{ lb. per ft.}$$

$$\frac{f_6}{f_M} = \frac{0{\cdot}1875}{2{\cdot}585}(19 - 14 \times 0{\cdot}805) = 0{\cdot}56;$$

$$w_{6x} = 4{\cdot}25 \times 0{\cdot}56 \times 193{\cdot}5 = 460 \text{ lb. per ft.}$$

$$\frac{f_7}{f_M} = \frac{0{\cdot}2188}{2{\cdot}585}(8{\cdot}875 - 7{\cdot}5 \times 0{\cdot}805) = 0{\cdot}241;$$

$$w_{7x} = 4{\cdot}25 \times 0{\cdot}241 \times 193{\cdot}5 = 198 \text{ lb. per ft.}$$

For the beams in the direction y, spanning 34 ft., the maximum bending moment occurs when both panels are loaded;

$r = 1$, $\alpha = \beta = 0{\cdot}4417$, $w_y = 0{\cdot}4417 \times 310 = 137$ lb. per square foot.

From *Table* I, $\frac{f_1}{f_M} = \frac{f_5}{f_M} = 0{\cdot}506$, and $\frac{f_2}{f_M} = \frac{f_4}{f_M} = 0{\cdot}869$.

$$w_{1y} = w_{5y} = 4{\cdot}25 \times 137 \times 0{\cdot}506 = 295 \text{ lb. per ft.}$$

$$w_{2y} = w_{4y} = 4{\cdot}25 \times 137 \times 0{\cdot}869 = 507 \text{ lb. per ft.}$$

$$w_{3y} = 4{\cdot}25 \times 137 = 582 \text{ lb. per ft.}$$

For these loads, the maximum bending moments on the beams in the direction x are

$$M_{1x(\max.)} = \frac{365 \times 25{\cdot}5^2}{8} = 29{,}600 \text{ ft.-lb.}; \quad M_{2x(\max.)} = \frac{655 \times 25{\cdot}5^2}{8} = 53{,}200 \text{ ft.-lb.},$$

$$M_{3x(\max.)}=\frac{850\times 25{\cdot}5^2}{8}=69{,}000 \text{ ft.-lb.}\ ;\ M_{4x(\max.)}=\frac{820\times 25{\cdot}5^2}{8}=66{,}600 \text{ ft.-lb.},$$

$$M_{5x(\max.)}=\frac{695\times 25{\cdot}5^2}{8}=56{,}500 \text{ ft.-lb.}\ ;\ M_{6x(\max.)}=\frac{460\times 25{\cdot}5^2}{8}=37{,}400 \text{ ft.-lb.},$$

$$M_{7x(\max.)}=\frac{198\times 25{\cdot}5^2}{8}=16{,}100 \text{ ft.-lb.}$$

The maximum bending moments on beams in the direction y are:
At the support:

$$\text{Beams 1 and 5, } M_{s1,5}=-\frac{295\times 34^2}{8}=-42{,}600 \text{ ft.-lb.}\ ;$$

$$\text{Beams 2 and 4, } M_{s2,4}=-\frac{507\times 34^2}{8}=-73{,}000 \text{ ft.-lb.}\ ;$$

$$\text{Beam 3, } M_{s3}=-\frac{582\times 34^2}{8}=-84{,}000 \text{ ft.-lb.}$$

At midspan:

$$M_{1y(\max.)}=M_{5y(\max.)}=0{\cdot}0703\times 295\times 34^2=23{,}900 \text{ ft.-lb.}\ ;$$
$$M_{2y(\max.)}=M_{4y(\max.)}=0{\cdot}0703\times 507\times 34^2=41{,}000 \text{ ft.-lb.}\ ;$$
$$M_{3y(\max.)}=0{\cdot}0703\times 582\times 34^2=47{,}300 \text{ ft.-lb.}$$

BIBLIOGRAPHY.

(1) Saliger. Der Eisenbeton. 5th ed., pp. 491–494. A. Kröner, Leipzig, 1925.
(2) Genttner. Der Eisenbeton-Trägerrost. "Beton u. Eisen," November and December, 1928.
(3) Habel. Näherungsberechnung von Trägerrostdecken für gleichmässig verteilte Vollbelastung. "Beton u. Stahlbetonbau," August, 1952.
(4) Bleich-Melan. Die gewöhnlichen und partiellen Differenzengleichungen der Baustatik, pp. 282 *et seq.* Springer, 1927.
(5) Müller. Panelled ceiling loaded by concentrated loads at the intersections of the beams. "Journal of A.C.I.," Vol. 21, December, 1949.
(6) Harband. Panelled ceiling loaded . . . etc. "Journal of A.C.I.," Vol. 22, September, 1950.

High-strength Notched Reinforcement.

WE have received the following corrections from Mr. A. Zaslavsky who supplied the data for the article on "High-strength Notched Reinforcement" in our number for February, 1959.

In the second paragraph on page 97 the proof stress of 25,000 lb. per square inch should read 52,000 lb. per square inch. In the last paragraph on page 98 the yield stress of 42,000 lb. per square inch should read 85,000 lb. per square inch.

Anticipatory Philosophy.

"THEIR (the U.S.S.R.) Seven-year Plan, which started in January 1959 and runs through 1965, calls for an 80 per cent. increase in over-all industrial production —for example, increasing steel production to 95,000,000 tons, electric power to 500 billion kilowatt-hours, cement to 440,000,000 barrels, and petroleum to 1·7 billion barrels. These are tremendous accomplishments."—A Doctor of Philosopy (Engineering) writing in the Bulletin of the American Society for Testing Materials.

Book Reviews.

"Hudson's Building Contracts." Revised by E. J. Rimmer and I. W. Duncan Wallace. (London: Sweet & Maxwell, Ltd. Price £4 4s.)

THIRTEEN years have passed since the previous edition was issued, and in that period the Conditions of Contract of the Royal Institute of British Architects have been revised. The eighth edition of this well-known and authoritative work will therefore be welcomed by the building industry. "Case law" is given relating to so many troubles that have arisen between builder and client or the agent of the client that such a voluminous work might appear to be entirely comprehensive, but new difficulties still arise. Perusal of such a volume is not a substitute for legal advice. But in this work are to be found the decisions of the courts on many problems that have arisen, and a knowledge of these decisions informs a builder of difficulties that may arise and may enable him to avoid them. The increasing use of arbitration in the settlement of disputes in the building industry is excellent in all respects except one, and that is that the arbitrator's awards are not published and in such cases no "case law" is available for the guidance of the industry or of the legal profession.

"Advanced Structural Analysis." By S. F. Borg and J. J. Gennaro. (London: D. Van Nostrand Co., Ltd. 1959. Price 56s. 6d.)

THIS U.S.A. book is intended for advanced students and for civil, mechanical, and aeronautical engineers, and attempts to go beyond what is ordinarily included in a text-book on the theory of structures in so far as recent developments such as the use of electronic computors are considered. A rigorous analysis of statically-indeterminate structures by the usual methods is given, in which deflections of beams and arches are dealt with in detail. Other chapters deal with the general theory of twisting of members of non-circular cross section but the application is mainly to metal structures, as is also the treatment of stresses in complex structures due to differences of temperature. A short chapter describes the plastic theory of design and the method of limit design in relation to beams; this chapter could usefully be extended.

Of particular interest is a long chapter on slabs and "shells". The mathematical theory of the action of flat plates is given, but the practical design of slabs as such by approximate, but sufficiently accurate, methods is not described. Thin "shell" structures are dealt with in detail and the comparison of two approximate and two so-called "exact" methods is particularly instructive. The exact methods are a general method based on the solution of the basic differential equations with assumed boundary conditions and the method given in the manual published by the American Society of Civil Engineers. The former method, called the English method by the authors, is that described in "Design of Cylindrical Shell Roofs", by J. E. Gibson and D. W. Cooper. The approximate methods are the beam theory and the membrane theory without modifications.

"Einflussfelder für die Hauptträgerschnittkräfte zweistegiger Plattenbalkensysteme." By Otto Köller. (Berlin: Wilhelm Ernst & Sohn. 1958. 6 D.M.)

A METHOD of determining the bending moments and shearing forces on the edge-beams of a bridge deck, the slab of which is supported along two edges only, is described in this booklet. The stiffness of the slab in both directions is taken into consideration. The method is claimed to give more accurate results and economical design than other methods applied to narrow slabs, but the calculations are much longer. Approximations are recommended for wide slabs, the torsional effects on which are not taken into account. For unsymmetrical loading the accurate method is claimed to result in considerable economy.

"Durchlaufträger, Rahmen und Platten." By J. Hahn. (Düsseldorf: Werner-Verlag. No price stated.)

THIS book for students describes the elementary applications of structural theory to beams, frames, foundations, and slabs spanning in two directions. Higher mathematics is avoided where possible, and within its scope the work is comprehensive.

FIG. 1.—A STANDARD BRIDGE OVER THE MOTORWAY.

The London–Birmingham Motorway.

THE London–Birmingham motorway includes two main parts, namely, the St. Albans by-pass road, which extends seventeen miles to the south-west of Luton, and the continuation northwards for 55 miles to the neighbourhood of Rugby. There are two branch roads from the St. Albans section, one being about five miles long to connect to the Watford by-pass road, and the other a short road to Luton. A preliminary description of these roads and details of some bridges on the northern section were given in this journal for March, 1958. Further details of the work, most of which is expected to be completed by the end of this year, are given in the following.

The main part of the St. Albans section comprises two reinforced concrete carriageways. The connections to existing roads and to Luton are of " flexible " construction comprising 4 in. of asphalt laid on rolled lean concrete similar to the construction of the main road of the Luton to Rugby section (see p. 342). The side roads comprise 2 in. of asphalt on 10 in. of rolled lean concrete. There are twenty-six reinforced concrete bridges on the main part of the St. Albans section and fourteen on the branch to Watford.

The section from Luton to Rugby comprises two main carriageways of " flexible " construction, roads to connect the motorway to existing roads, service areas, 125 reinforced concrete bridges, three steel bridges to carry railways over the motorway, and four reinforced concrete viaducts to carry the motorway over the rivers Great Ouse, Lovat, and Nene. There are also ninety-two culverts some of which are 300 ft. long and up to 20 ft. wide. The work is in four main divisions of lengths of 12·5, 11·9, 12·3, and 17·7 miles respectively.

Fig. 2.—A Batching and Mixing Plant.

Bridges on the St. Albans By-pass Road.

On the St. Albans by-pass road there are thirty road bridges, two railway bridges, a footbridge, three subways for pedestrians, three large culverts, and a bridge over the River Colne. Twenty-six of these structures were designed by the Hertfordshire County Council. The other fourteen, which are similar to the standard bridges on the London to Rugby motorway and were designed by Sir Owen Williams & Partners, the consulting engineers, are rigid-frame structures of one or two spans, the decks comprising slabs without beams and with solid concrete parapets.

The bridges designed by the Hertfordshire County Council are of two principal types. Bridges over the motorway have generally four spans supported on three intermediate trestles and on the slopes

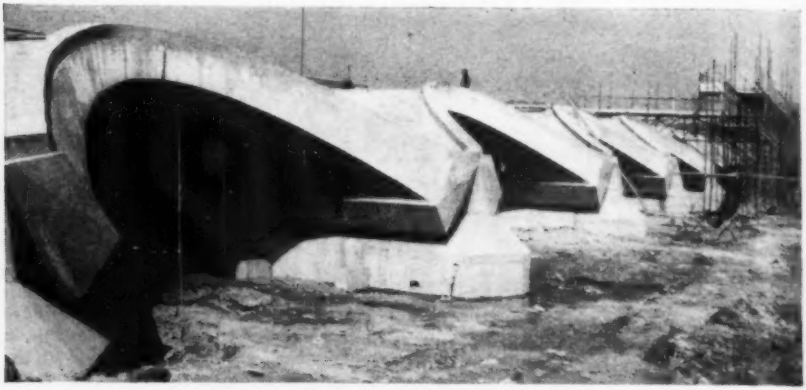


Fig. 3.—Viaduct over the River Ouse.

of the cuttings or embankments carrying the connecting roads. Bridges carrying the motorway have generally three spans; the intermediate supports comprise two rows of double sloping columns and the ends are supported on the embankments. The decks of both types are thin slabs. There are also single-span bridges having plain concrete abutments faced with brickwork, and one bridge of two spans of similar construction. The parapets of most of these bridges will be steel railings. Four of the bridges have piled foundations.

Bridges on the Luton–Rugby Road.

The bridges on the Luton–Rugby road were, in general, constructed in advance of the adjacent parts of the motorway. Priority was given to viaducts and railway and canal bridges, since these required more time for erection, and to bridges carrying the motorway in order to avoid delay in constructing the embankments leading up to, and in some cases over, such structures. The bridges were also used to transport plant, materials, and other constructional traffic. A standard two-span bridge over the motorway is shown in *Fig.* 1. There are sixty bridges of this type and nineteen four-span bridges of similar design.

The erection of each major bridge is dealt with as a separate constructional problem by a team of men equipped with the scaffolding, shuttering, and mixing and placing plant. When a bridge is completed the team with its equipment moves to the site of the next structure. The amount of shuttering provided is sufficient to enable concrete to be placed continuously to the full capacity of the plant.

The batching and mixing plant (*Fig.* 2) for each bridge comprises a drag-line or hand-guided scraper for transferring aggregates from stock-piles to the weigh-batchers, a batching plant, and one or two mixers up to 21/14 cu. ft. capacity. Cement is delivered by road in pressure tanks and stored in silos of 10 to 30 tons capacity. The concrete is transported

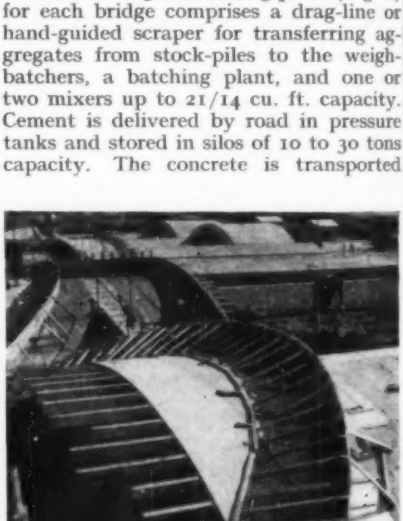


Fig. 4.—Shuttering for Fascia of Arch.

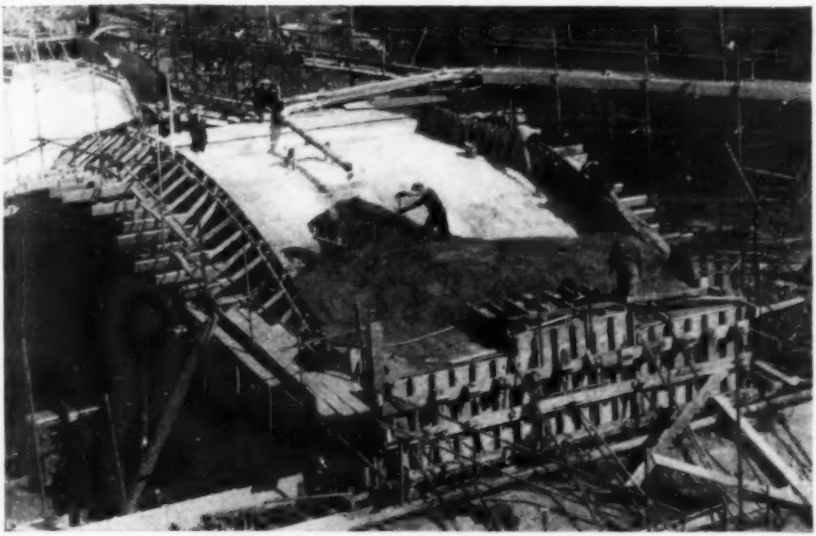


Fig. 5.—Placing Concrete for an Arch.

by various means depending on the distance, the relative levels of the mixing plant and the deck of the bridge, and other site conditions, and include small dumpers, skips on mono-rails, pumps, and skips transported by cranes. The shuttering for the decks of standard bridges over the motorway comprises timber boards on longitudinal timber bearers supported on steel props. The accompanying illustrations show some of the bridges in course of construction.

Fig. 3 shows one of the four-span viaducts over the Great Ouse. The shuttering for the arches is shown in *Fig.* 4.

In *Fig.* 5 pumped concrete is shown being placed for the arches of the two-span viaduct over the same river; the temporary bridge carrying the pipe-line is seen in the left background.

The viaducts over the rivers Lovat and Nene are of wall-and-slab construction. Travelling centering, comprising mainly steel beams on shutter-prop trestles, was used for four bridges of similar construction over canals.

The centering shown in *Fig.* 6 is for a bridge crossing a canal on a skew. Concrete was placed from a skip transported by a crane. The deck was concreted in narrow widths for the complete span of the bridge at each position of the centering. The side shutters at construction joints between successive widths were corrugated steel sheets laid with the corrugations horizontally to form a key.

In the case of the bridge in *Fig.* 7, which is a three-hinge arch of plain concrete and for which a travelling centre was also used, the key at construction joints between adjacent transverse sections was formed by deep indentations in the face of the arch. In this case the transverse sections were not concreted consecutively. Immersion vibrators were used to compact the concrete, which was sufficiently stiff to make it unnecessary to use top shutters near the haunches.

Precast construction (*Fig.* 8) is used for parts of bridges carrying the motorway over railways. The abutment walls and the foundations for the row of columns were cast in place in 1 : 2½ : 4½ concrete. The circular reinforced concrete columns and the capping beams, of 1 : 1 : 2 concrete, were cast in place, and the precast beams at a right angle to the railway were then erected (*Fig.* 8). A slab of 1 : 2 : 3 unreinforced concrete was laid on the sloping surface formed by the beams. Earth filling was then deposited on the sloping slab to act as a counterweight, and was considered to form a continuation of the embankment carrying

Fig. 6.—Mobile Support for Shuttering.

the road. The filling is retained at the sides by walls of plain concrete. The deck of the bridge comprises precast reinforced slabs supported on the ends of the cantilevered beams and placed parallel to the direction of the road. A layer of 1 : 2 : 3 reinforced concrete was placed on the precast slabs. The level of the earth filling was then brought up to the level of the top of the deck. The design of this bridge is illustrated in this journal for March, 1958.

In addition to the bridges and viaducts mentioned in the foregoing, the motorway is carried on twenty-nine slab bridges of one span, seven arch bridges, one bridge of two spans, and a wall-and-slab bridge over a stream. The bridges are tested by loading them with lorries containing aggregates. The materials in the bridges include about 136,000 cu. yd. of plain concrete, 150,000 cu. yd. of reinforced concrete, and 12,000 tons of reinforcement.

Reinforced Concrete Carriageways.

The carriageways (*Fig.* 9) on the St. Albans by-pass road are each 38 ft. wide. South of the bifurcation towards London and Watford each carriageway

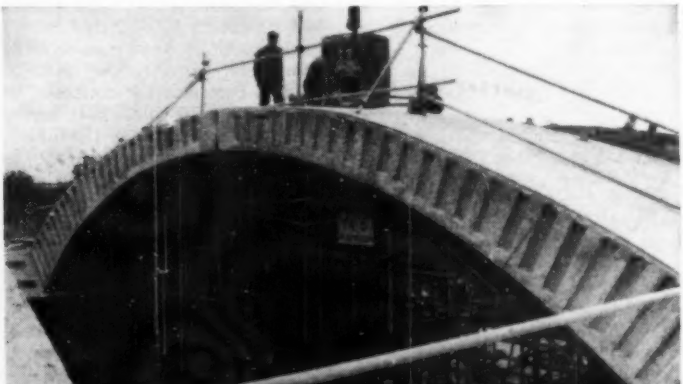


Fig. 7.—Transportable Shutter for Arch.

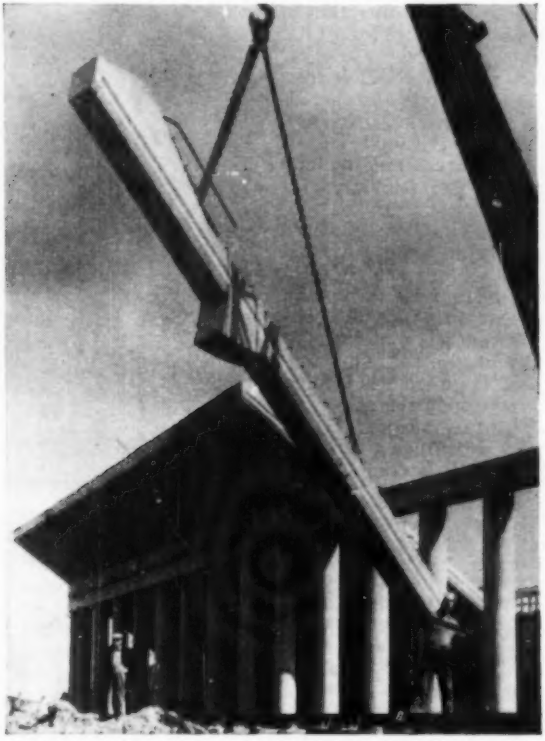


Fig. 8.—Placing Precast Beams on Columns and Sills Cast in Place.

is 26 ft. wide. The construction comprises an 11-in. slab reinforced near the top with welded steel mesh weighing 9·32 lb. per square yard. The slab, which is constructed in two courses, is laid to a cross-fall of 1 in. in 4 ft. on a 7-in. granular base. There are white strips 1 ft. 6 in. wide at each side. A central longitudinal joint is formed by a 3-in. triangular wooden fillet embedded in the bottom and a saw-cut ¼ in. wide in the top. Dowel bars ¼ in. in diameter and 3 ft. 6 in. long are provided transversely at 2 ft. centres between the fillet and the saw-cut. Details of the side drainage, central reserve, and verges are given in *Fig.* 9.

Concrete for the main part of the road is mixed in three plants and placed and finished by means of three sets of concreting machines two of which are 26 ft. wide and one 12 ft. wide. Each set of machines comprises a spreading and compacting machine and a finishing machine (*Fig.* 10) which was designed by the contractors for this road. Concrete is brought from the mixing plant in side-tipping lorries from which it is discharged into the hopper of the spreading machine. The finishing machine has a heavy oscillating beam which is mounted on bogies and which removes excess concrete and smooths out any irregularities. The surface of the slab is brushed by hand-brooms by men working from a light bridge spanning the road, and a curing membrane is then sprayed on to the surface. About 900 ft. of carriageway can be laid in a day by the machines 26 ft. wide and about 1300 ft. by the machines 12 ft. wide. The index of irregularity of most of the surface is less than 30 in. per mile as measured by the profilometer of the Road Research Laboratory. The

irregularity in a longitudinal direction in 10 ft. of road is generally less than $\frac{1}{32}$ in.

The carriageways of the branch road to Watford are being concreted by spreading, compacting, and finishing machines, the concrete for which is supplied by a mixer travelling beside the road-making machines (*Fig.* 11). The foundation of the road comprises a layer of broken concrete about 7½ in. thick, the surface of which is covered with sand laid and levelled by a spreading machine to a tolerance of ¼ in. The sand is covered by a sheet of polythene on which the concrete is laid. The concrete materials are batched dry at a central plant and transported to the concrete mixers in tipping lorries having three compartments. The concrete is loaded into the trough of the spreading machine, which has a capacity of about 2½ cu. yd. The machine travels on the side forms and spreads an 8-in. layer of concrete on which the reinforcement is laid. An additional layer 3 in. thick is then deposited. The compacting and finishing machines follow the spreading machine. The finishing operations, which include brushing by hand broom and spraying with a membrane curing compound, are carried out under a canvas cover. Expansion joints are cut in the hardened concrete by a mechanical saw. The equipment shown in *Fig.* 11 has, on one occasion, laid 1200 ft. of carriageway, 26 ft. wide, in eleven hours.

The total amount of concrete in the roads and bridges of the St. Albans by-pass road, including the branch to Watford, will be about 61,500 cu. yd. and there will be about 4700 tons of reinforcement.

"Flexible" Roads.

The "flexible" carriageways of the Luton to Rugby part of the motorway and subsidiary roads in connection with the St. Albans part comprise primarily a layer of compacted lean concrete 14 in. thick laid on a 6-in. granular base. The concrete is covered with 2½-in. of asphalt with large aggregate upon which is laid a 1½-in. hot-rolled wearing course of finer asphalt.

The width of each carriageway is 36 ft. and at the sides of both carriageways there is a concrete haunch 12 in. wide. For the construction of the carriageways, verges, and central reserve, the Luton to

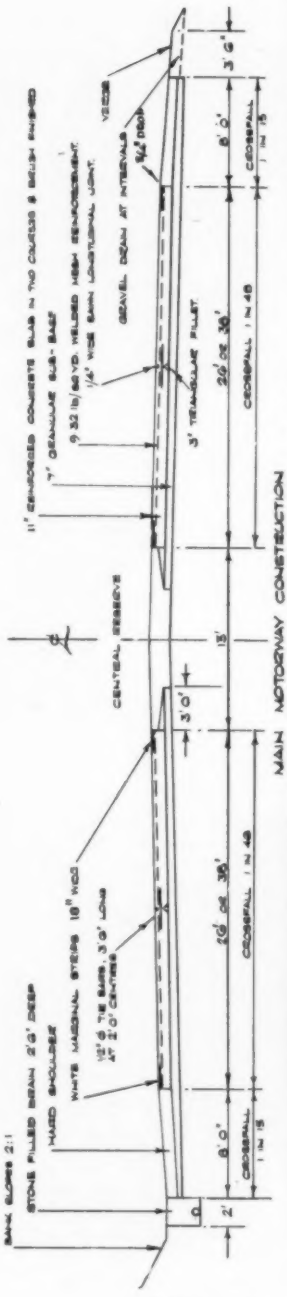


Fig. 9.—Cross Section of St. Albans By-pass Road.

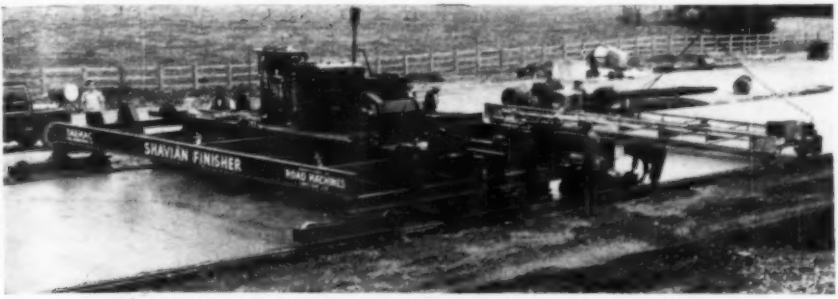


Fig. 10.—Road-making Machine: St. Albans By-pass Road.

Rugby road is divided into lengths of about 3 miles each. The lean concrete is a mixture of one part of cement to 18 parts of fully-graded aggregate. No water is added, there being sufficient moisture in the aggregate to cause the cement to set.

Thirteen concrete mixing plants are installed along the length of the road. Nine of these plants have twin-drum mixers which are charged from a standard weigh-batching plant altered for this work. At each batching plant there are two elevated bins. One bin contains the aggregate and is loaded with a dragline excavator of $\frac{3}{4}$ cu. yd. or $\frac{7}{8}$ cu. yd. capacity. The other bin has a capacity of 35 tons of cement and is filled from a pressure-tank on a lorry. Each plant has a capacity of 3000 cu. yd. in an ordinary working week. There are also four continuous mixing plants.

The concrete is transported from the mixers in tipping lorries to paving machines (*Fig.* 12), which spread, compact, and level the material in two layers each 7 in. thick. Final consolidation and finishing are by rolling with tandem-rollers weighing 6 tons to 8 tons and small vibratory rollers. The concrete is laid in widths of 16 ft. and is sufficiently stiff for the edges, which are smoothed by a plate on the side of the machine, to stand vertically without fixed side forms.

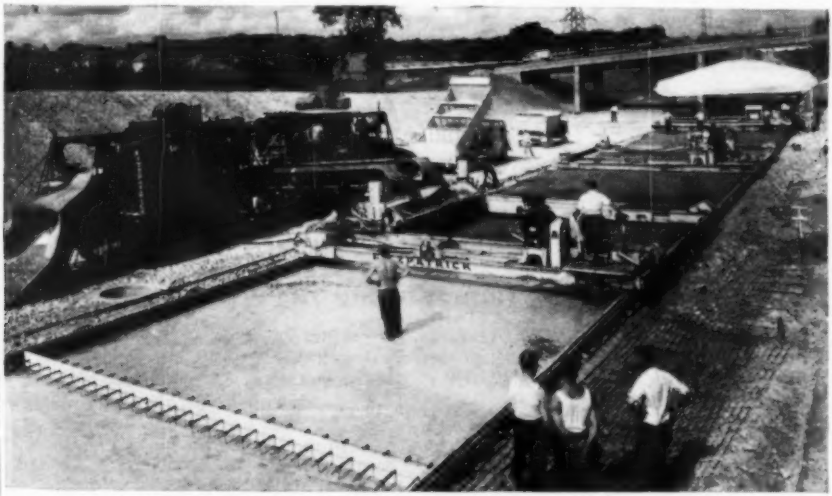


Fig. 11.—Road-making Machines: Watford Branch Road.

Fig. 9.—Cross Section of St. Albans By-pass Road.
MAIN MOTORWAY CONSTRUCTION

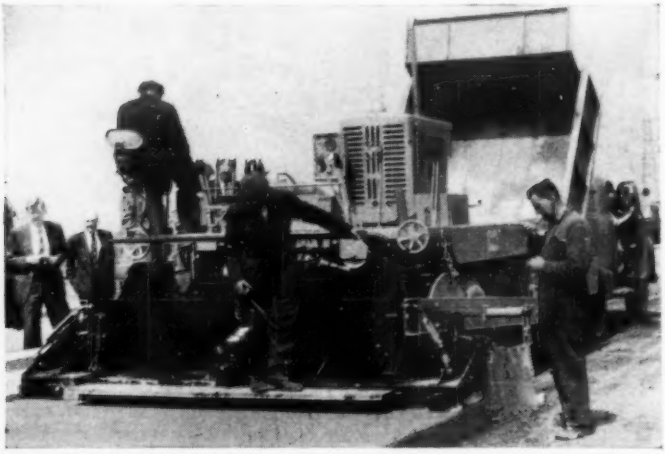


Fig. 12.—Machine placing Lean Concrete for "Flexible" Road.

(An American method of concreting roads without fixed side forms is described in this journal for February, 1958.) There are about 900,000 cu. yd. of lean concrete in the carriageways and about 115,000 cu. yd. of reinforced concrete in the haunches.

The haunches are 12 in. wide at the top and 18 in. at the bottom and are of normal concrete with two reinforcement bars near the top. Steel shutters are used for the sloping sides. Dowelled contraction joints are provided at intervals. A layer of white concrete 1 in. thick is placed on top immediately after placing the ordinary concrete. The white concrete comprises crushed calcined flint aggregate and white cement to which has been added 5 per cent. by weight of titanium oxide in accordance with the specification of the Ministry of Transport.

The motorway is being constructed for the Ministry of Transport and Civil Aviation. The Hertfordshire County Council (County Surveyor, Lt.-Col. C. H. ffolliott) are the agents of the Ministry for the St. Albans section. The contractors for the main part of the St. Albans section are Tarmac Civil Engineering Ltd. The Watford branch is being constructed by Messrs. Holland & Hannen and Cubitts (Great Britain), Ltd., Messrs. Fitzpatrick & Sons (Contractors), Ltd., and Messrs. Lehane, Mackenzie & Shand, Ltd.

The consulting engineers for the Luton to Rugby section and for some of the bridges on the St. Albans section are Sir Owen Williams & Partners. The contractors are Messrs. John Laing & Son, Ltd., who supplied the photographs of *Figs.* 1 to 8 and 11.

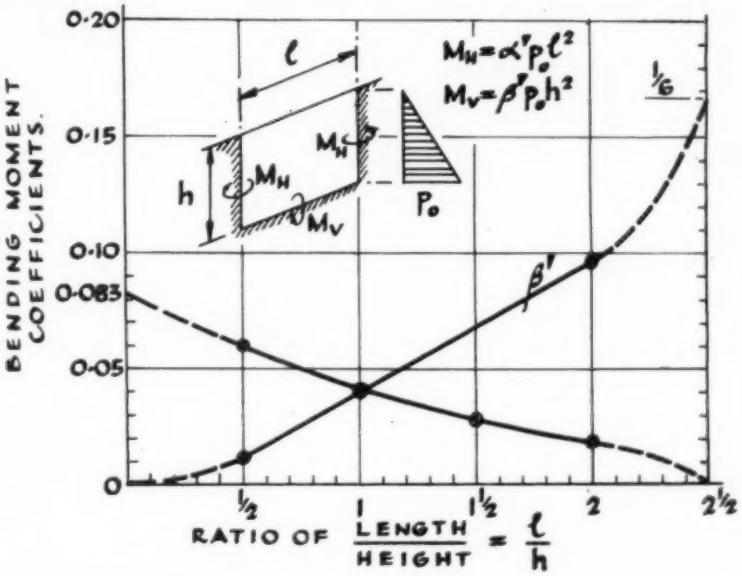
Lightweight Concrete for Bridge Decks.

CONCRETE made with expanded-shale aggregate was used for the deck of the Forest City bridge over the River Missouri in South Dakota, U.S.A. The deck is 4575 ft. long by 26 ft. wide and is 5½ in. thick. The concrete was mixed in the proportions of 1210 lb. of fine aggregate, 404 lb. of coarse aggregate, 634 lb. of cement, and 29 Imperial gallons of water. The average compressive strength was 5626 lb. per square inch and the density 101 lb. per cubic foot.

Expanded-shale aggregate was also used for the deck of the central span of a bridge near Chicago. The concrete contained 750 lb. of cement per cubic yard, and the compressive strength was about 5000 lb. per square inch.

Walls of Rectangular Tanks Unsupported at the Top.

THE bending moments on walls of rectangular open tanks which span horizontally between counterforts and are monolithic with the bottom of the tank but unsupported along the top can be calculated by dividing the pressure on the wall into two parts, one of which is assumed to produce bending moments horizontally and the other vertically. The division can be made geometrically or, less arbitrarily, by equating the deformations at any point due to the bending in the two directions. The mathematics may be complex in the latter case, but various methods have been devised to avoid the complexity in practical design. Such a method was given by J. F. Büchi in "Beton und Eisen" for April 20, 1938, and was noted in "Concrete and Constructional Engineering" for July, 1951. The accompanying graph is based on the method devised by Mr. Büchi and shows the bending-moment coefficients for various ratios of length to height of such walls.



In accordance with this method the maximum bending moment M_H at the vertical edges of a panel are $\frac{\alpha p_0 l^2}{12}$ on an imaginary horizontal strip of unit width, if p_0 is the pressure at the bottom of the wall and l is the length of the panel. This bending moment occurs on a strip at a height of one-quarter to two-thirds of the depth h of the tank; the greater the length compared with the depth the higher the point of maximum bending moment.

The bending moment, horizontally, at the middle of the wall is about half that at the corners at the same height.

The maximum bending moment M_v on a vertical strip of unit width is $\frac{\beta p_0 h^2}{6}$ and occurs at the middle of the bottom edge of the panel.

The coefficients α and β depend on the ratio of l to h and are given by $\alpha = \left(1 - \frac{m}{6{\cdot}2}\right)^4$ and $\beta = \frac{m - 0{\cdot}3}{3}$.

The curves give values of $\frac{\alpha}{12}(= \alpha')$ and $\frac{\beta}{6}(= \beta')$ for various ratios of $\frac{l}{R}$. This method is not applicable to panels having ratios of $\frac{l}{h}$ less than half or greater than two. A tall narrow panel, that is a panel having a ratio less than half, tends to span entirely horizontally, and the maximum bending moment occurs near the bottom and will approach $\frac{p_0 l^2}{12}$, that is $\alpha = 1$, and $\alpha' = \frac{1}{12}$. A long low panel, that is a panel having a ratio greater than two, tends to act entirely as a vertical cantilever, the maximum bending moment at the bottom being $\frac{p_0 h^2}{6}$, that is $\beta = 1$ and $\beta' = \frac{1}{6}$.

The curves are drawn so that they tend to approach these limiting values of $\alpha' = \frac{1}{12}$ and $\beta' = \frac{1}{6}$ but conform to $\frac{\alpha}{12}$ and $\frac{\beta}{6}$ respectively for the intermediate range of $\frac{l}{h} = \frac{1}{2}$ to 2.

The Shearing Strength of Reinforced Concrete.

RESEARCH on the shearing strength of reinforced concrete described in "Building Research, 1958" (H.M. Stationery Office. Price 5*s.* 6*d.*) is intended to provide data for a revision of B.S. Code of Practice No. 114. The work in the first stage, dealing with beams without shear reinforcement, is now complete. The effects of a number of variables have been investigated, including the distance of the load from the support, the manner of loading the beam (that is whether on the top surface or through secondary beams framing into the sides of the beam), the characteristics of the reinforcement (plain round mild-steel bars or cold-worked deformed mild-steel bars), the amount of reinforcement, the strength of the concrete, the shape of the beam, and continuity over supports.

One conclusion is that the load at which diagonal cracking occurs, and not the load causing collapse, should preferably be used as the criterion of failure. When a beam is loaded at the top surface its ultimate strength may, under certain conditions, be as much as three times the load at which diagonal cracks form. However, when diagonal cracks have formed their subsequent development is such that it would probably be unwise to place much reliance on the capacity of the beam to carry further load. In the case of beams loaded through secondary beams framed into the sides collapse occurs at, or soon after, the occurrence of diagonal cracking.

Another conclusion is that the present recommendations of the Code are not wholly satisfactory. Several of the variables have an effect on the shearing strength but are ignored in the Code. It is usual for design rules to be based on tests carried out under static loads, but structural members may be subjected to repeated loading. Tests show that when a pulsating load is superimposed on a load representing the dead load, the maximum load that can be sustained without causing diagonal cracks is about 70 per cent. of that at which such cracks are caused by a single loading.

Precast Cantilevered Prismatic Roof.

NEW HEADQUARTERS OF THE AMERICAN CONCRETE INSTITUTE.

THE office building (*Fig.* 1) for the American Concrete Institute in Detroit is a single-story structure 97 ft. long and about 37 ft. wide with a semi-basement. The most interesting structural feature is the double-cantilevered roof (*Figs.* 2 and 3) which has a width of about 48 ft. and is constructed of precast prismatic members (folded plates) supported on and cantilevering 19 ft. from each of the two 8-in. walls of a longitudinal corridor. These walls and the walls of the basement are of reinforced concrete cast in place. Deep beams of X-section in plan, with openings for roof lights, form the tie between opposite roof members.

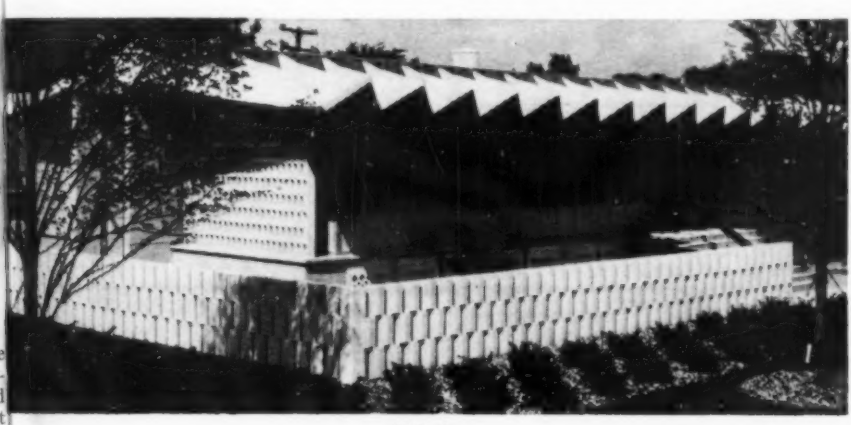


Fig. 1.

The roof members, which are about 24 ft. long and 4 ft. 4 in. wide, change in cross section from a vee-shape at the eaves to an inverted vee at the supporting wall. Precasting on the site (*Fig.* 4) was adopted because it was considered to be cheaper, and by casting the members upside down sharper edges were obtained than if the slabs had been cast in place. The members were cast in plastic-faced plywood moulds, three of which were provided. A prototype mould was prepared to serve first as a pattern for cutting the plywood and then as a jig for assembling cages of reinforcement. The construction programme required two members to be cast daily. Therefore the concrete was required to have a compressive strength of at least 1500 lb. per square inch at 24 hours and 4000 lb. per square inch at 28 days. The average strength at 28 days was 5100 lb. per square inch. Placing the concrete was difficult and slow because of the congestion of reinforcement and the presence of inserts in the slabs, which are $3\frac{1}{2}$ in. thick. The concrete contained 585 lb. of ordinary Portland cement per cubic yard. The aggregate was $\frac{3}{8}$-in. gravel, and a compound to retard setting was used. With an average initial slump of 3 in., setting was retarded for about $1\frac{1}{2}$ hours when the temperature of the concrete was up to 95 deg. F. The average compressive strength at twenty-four hours was 2100 lb. per square inch. The members were cured, first by covering with wet burlap for twenty-four hours, and then by spraying with a liquid which formed a white coating.

After curing, the members were turned over so that the top face could be finished. Eye-bolts for lifting the members were threaded into sockets embedded in the concrete. The members were numbered and the overall width of each measured so that a sequence of erection could be followed to avoid cumulative errors in the dimensions of the finished structure.

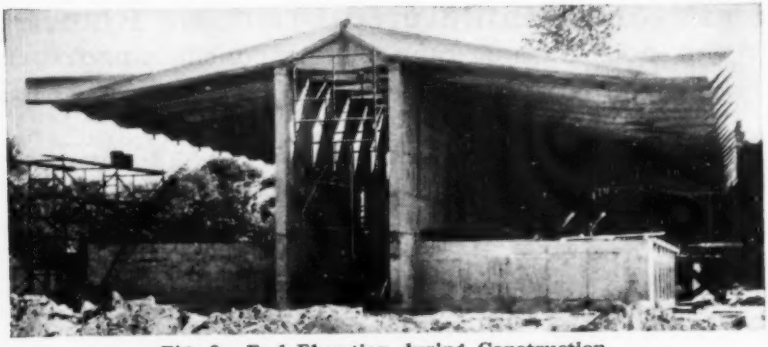


Fig. 2.—End Elevation during Construction.

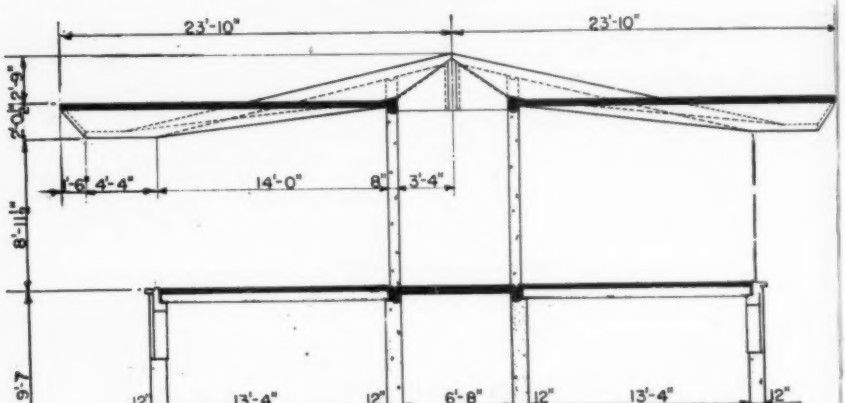


Fig. 3.—Transverse Section.

Fig. 4.—Casting Roof Members.

The erection procedure was to transfer the members by crane from the casting yard to a position alongside the structure and then to lift each one into place with one end on the corridor wall and supported intermediately on a scaffold (*Fig.* 5) on the inset line of the windows. Two 4-in. steel shims on the walls and a point support on a scaffold provided a stable three-point bearing that permitted adjustment of the adjoining members.

The joint between adjacent members was formed by welding together abutting $\frac{3}{8}$-in. steel strips which were cast in each edge of the members at the time of casting. The connections to the wall are formed by welding to steel dowels projecting from the top of the wall. Non-shrinking mortar was used to fill the short central gap and for the bearing on the wall. A sharp edge was formed on the joints above the longitudinal welds between the members by patching with mortar containing a polyvinyl-acetate emulsion. A rise of $1\frac{1}{4}$ in. was provided at the inset line of the windows. Immediately after erection, the average deflection at this line was $\frac{1}{16}$ in. The heads of the windows are provided with a sliding joint to allow for additional deflection due to live load and creep. The deflection at the ends of the cantilevers, when the scaffold was removed, was about $\frac{3}{8}$ in. due to dead load. Plastic flow and additional dead load caused an increase to about $\frac{3}{4}$ in.

The cantilevered members were designed by the ordinary beam theory, that is assuming a straight-line variation of strain because the proportions of the members made more accurate analysis as prismatic slabs unnecessary. In the direction at right-angles to the direction of the cantilevers, the members are reinforced as continuous slabs supported at the junctions and reinforced for positive and negative bending moments which might result from non-uniform and concentrated loads. They are also reinforced to provide resistance to the entire shearing force. The slabs are relatively thick and the stresses near the free edges are not severe. The slabs are exposed to changes of external temperature with resultant bending.

Lightweight concrete would have been desirable for this roof since it has greater insulation value, but was not used because of its greater cost. Insulation and roof coverings were also considered to be too expensive for a roof of such complicated shape. The soffit of the roof was boarded and plastered.

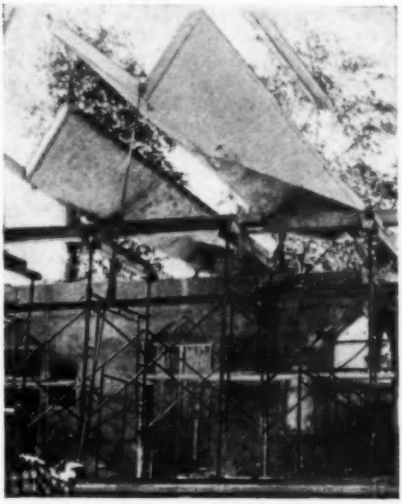


Fig. 5.—Erecting Roof Members.

The architect was Mr. Minoru Yamasaki, the engineer was Mr. Charles S. Whitney, and the contractors Messrs. Pulte-Strang, Inc.

British Standard for Concrete Mixers.

THE principal alterations in the revised edition of British Standard No. 1305 (1959), "Batch-type Concrete Mixers", are the omission of particulars of 2-cu. ft. tilting mixers and 84-cu. ft. and 112-cu. ft. non-tilting mixers, and amplification of the requirements for water tanks and the fittings for such tanks. Additional mechanical details regarding power-loaders are given. In other respects the principal requirements are as in the 1946 edition.

Film on Prestressed Concrete.

A FILM strip showing photographs of some of the prestressed concrete structures described at the third congress of the Fédération Internationale de la Précontrainte is available from the Cement and Concrete Association, 52 Grosvenor Gardens, London, S.W.1, at £2 10s. for the first copy and £2 for each subsequent copy.

Rubber Bearings for Bridges.

THE illustrations show a type of rubber bearing used in Pelham bridge, Lincoln, and approved by the Ministry of Transport for this bridge and also for a bridge to be built as part of the extension of Cromwell Road, London.

Pelham bridge is nearly a quarter of a mile long including the approaches, with dual carriageways and pavements. The approach spans are in reinforced concrete and the central spans are welded steel girders with a concrete deck, and these spans are to be supported on rubber bearings.

At present two types of bearings are available to support loads of 50 and 100 tons respectively. In appearance the bearings are blocks of rubber, the larger blocks measuring 24 in. by 16 in. by $7\frac{1}{8}$ in. deep, and the smaller 11 in. by 16 in. by $7\frac{1}{8}$ in. deep. The rubber is interleaved with steel plates (*Fig.* 1) having a minimum cover of $\frac{1}{4}$ in. of rubber around their edges. The interleaved steel is bonded by the brass plating process. With a view to increasing the durability of the bearings, they are made of special natural rubber embodying anti-ozonants and anti-oxidants ; the Shore hardness is 65 to 70.

Figs. 2 and 3 show the effect on the bearing of direct loading and shearing forces. While a shearing strain of about 45 deg. can be withstood for short periods,

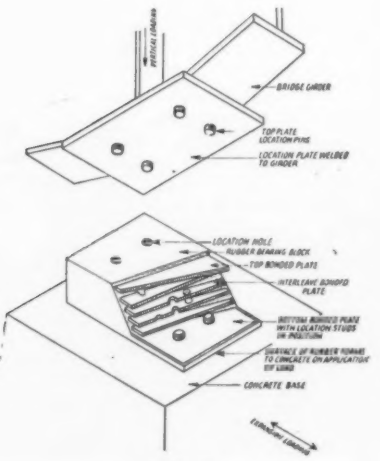


Fig. 1.

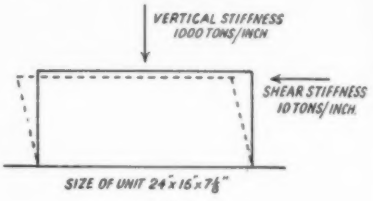


Fig. 2.

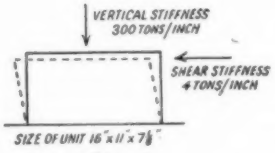


Fig. 3.

it is desirable for long life to limit it to 10 deg., corresponding to an expansion of about $1\frac{1}{4}$ in. in the case of bearings 7 in. deep. Where the expansion is less the depth of the bearing can be reduced ; both the shearing and the compression are decreased nearly linearly as the depth is decreased. It is stated that the force required to cause a shearing strain of 10 deg. is equivalent to that exerted by a sliding bearing with a coefficient of friction of 0·07.

The resistance to shearing is the same in any direction in the horizontal plane. This allows a skew bridge freedom to expand across its width relative to the abutments as well as along the length of the span, but the structure would have a low natural frequency of vibration in the horizontal plane and some provision may have to be made for this in the design ; this problem is being considered.

The compression of the bearings is such that for most bridge work they can be regarded as pinned supports, the actual rotational stiffness for the larger bearing being 1,800 tons-ft. per radian. In the case of Pelham bridge the rotations are 12 minutes under the Ministry of Transport standard highway loading, and the couple exerted by the bearings mounted lengthwise to the bridge is 6 tons-ft. This may be compared with a theoretical figure of 15 tons-ft. for an unlubricated steel-pin bearing.

In some recent tests bearings designed

for static and live loads totalling 130 tons were tested to 200 tons, when the deflection was 0·2 in. for each bearing, or one-thousandth of an inch per ton.

The consulting engineers for Pelham bridge are Messrs. W. S. Atkins & Partners, who collaborated with the Andre Rubber Co., Ltd., and the British Rubber Producers' Research Association in producing the bearing described.

A Tank to contain 600,000 Gallons of Petrol.

A TANK of unusual shape has recently been constructed in France for storing petrol for Le Service des Essences de l'Armée Française. The tank is underground with a cover of 8 ft. of earth in two layers between which is a concrete slab 1 ft. 6 in. thick. The capacity of the tank is about 600,000 gal. and it is designed for an internal pressure of 830 lb. per square foot and an internal suction of 40 lb. per square foot in addition to the weight of the covering materials. The following is abstracted from an article by M. Jean Courbon in the French journal " La Technique Moderne Construction ".

The walls of the tank are annular (*Fig.* 1), the roof is domed, and the base is an inverted dome. The principal dimensions are: External wall: mean radius 35 ft., thickness 12 in., height 18 ft. Roof: thickness 6 in., rise 5 ft. 9 in. Central pillar: external radius 5 ft.,

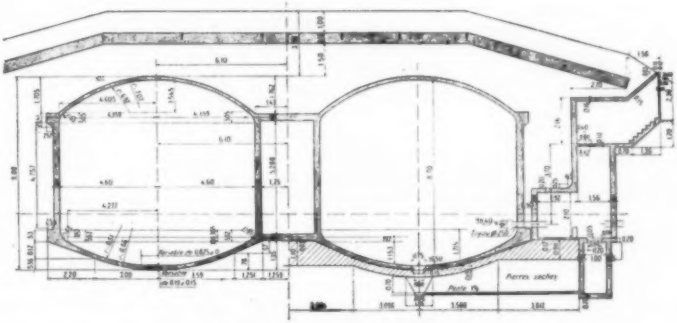


Fig. 1.—Cross Section.

Fig. 2.—Shuttering for Roof.

height 18 ft. Base : thickness 6 in., rise 4 ft. 6 in. The external wall is prestressed by 50 vertical cables each exerting a force of 90 tons, and by 28 horizontal cables, each in the form of a semi-circle and disposed as follows : Six cables tensioned to 55 tons in the tie-beam around the roof, fourteen cables tensioned to 65 tons in the wall, and eight cables tensioned to 90 tons in the lower tie-beam. Other parts of the structure are reinforced with steel bars. The base of the tank is supported by a concrete bed with a least thickness of 12 in. (shown hatched in *Fig.* 1), but between the two layers of concrete is 1½ in. of tar-macadam to prevent adhesion between the two surfaces and thus allow expansion and contraction of the tank. The sub-soil is sand and gravel.

Timber shuttering was used throughout (*Fig.* 2), an entire wall being shuttered at once. The shutters for the roof were made in panels and supported by timber frames spanning between the walls, thus eliminating propping. A plywood lining was used against all interior surfaces. The concrete is a 1 : 2 : 3½ mixture.

Prestressing was by the Chalos system. This method utilises cables consisting of two layers of high-tensile wires wound in opposing directions around a core formed of short cylindrical pieces of steel with flexible joints. The wires are tensioned against the core and are anchored by the device shown in *Fig.* 3. The assembly, already tensioned, is placed in the shutters and when the concrete has hardened the core is extracted through the hole in the centre of the anchor.

Considerable care was taken to achieve the greatest degree of watertightness. First the tank was washed with a 10 per cent. solution of hydrochloric acid to remove all traces of free lime from the cement on the surface, the acid being removed by scrubbing with water. The inside was then sprayed with two coats of a proprietary material, a layer of glass, and five more coats of the waterproofing material. Finally the whole was covered with a layer of aluminium sheet 0·12 mm. thick. Three tests were carried out with the tank filled with water at a pressure of 830 lb. per square inch. During the first test, which lasted twelve days, 130 gal. of water were lost ; the following tests lasted ten days each and the losses were 30 gal. and 45 gal. respectively from an original volume of 600,000 gal.

The tank was designed and constructed by the Société des Grands Travaux de Marseille.

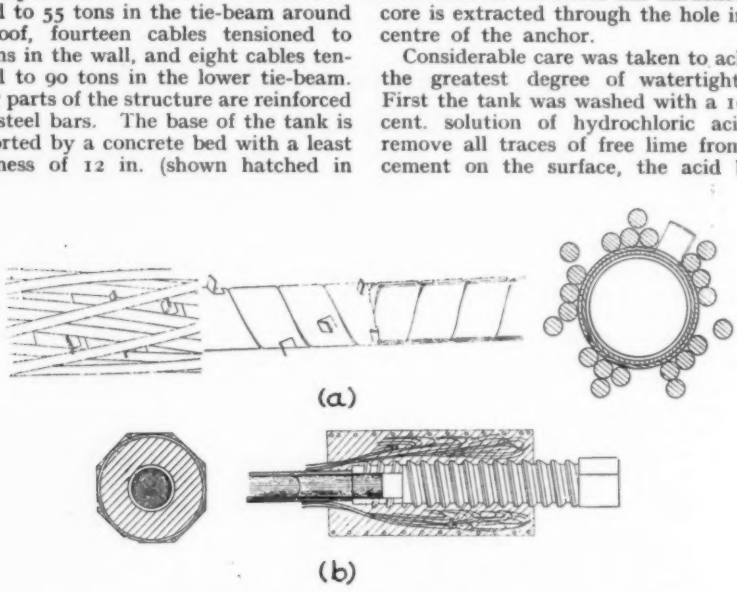


Fig. 3.—Prestressing Details.

An Experimental Road in Huntingdonshire.

THE design and construction of an experimental carriageway, 2¼ miles long, at Alconbury Hill, Huntingdonshire, are described in "Road Research 1957" (H.M.S.O., price 5*s.* 6*d.*), from which the following notes are abstracted.

The experimental section was laid in the year 1957 and forms part of the northbound carriageway. The experiment was the largest so far undertaken in Great Britain, and was designed to provide information about the behaviour of many forms of construction when used on a weak clay subsoil and subjected to heavy traffic. One mile of the carriageway is of concrete, and 1¼ miles are of flexible construction. The thicknesses and strengths of the concrete, the amounts of reinforcement and the types of base are shown in *Fig.* 1, and details of the flexible construction are shown in *Fig.* 2. Instruments have been installed to measure pressures and deformations and changes in moisture conditions. Non-destructive testing was used during construction, in conjunction with conventional sampling and loading tests.

The subsoil at formation level varies from boulder clay to Oxford clay. The concrete section is intended to determine the smallest thickness of concrete necessary for the conditions at the site, and the extent to which it is affected by the amount of reinforcement, the type and thickness of the base, and the strength of the concrete. In the flexible section the effect of the thickness of the sub-base, the type and thickness of the base, and the type and thickness of the surface will be studied.

The materials used are as follows.

SUB-BASE.—A local fine sand.

BASES:

Uncoated stone.—Graded slag mixed wet. Compacted with an 8-ton roller.

Coated stone.—(a) Tarmacadam to B.S. 802, Table 1 (b). Nominal size 1½ in.; tar content 2 to 3 per cent., viscosity 52 deg.

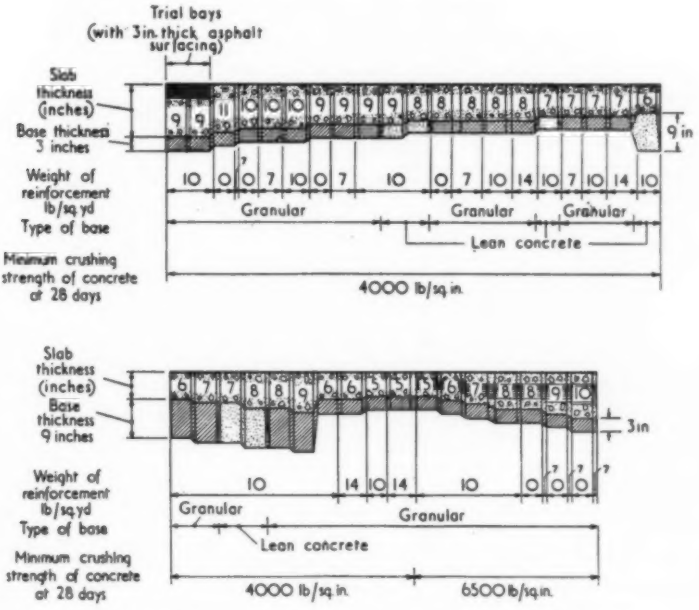


Fig. 1.—Arrangement of Experimental Sections.

equiviscous temperature. (b) Close-textured tarmacadam with 5 per cent. of tar.

Rolled asphalt.—To B.S. 594, Table 11, Schedule 4; 65 per cent. stone; asphalt cement to Table 6, column 1; asphaltic bitumen 40–60 penetration; stone 1 in. maximum size.

Lean concrete.—Aggregate 1½ in. maximum size; proportions 1 to 15 by weight; water content 5 per cent. of weight of material; compacted with 17-cwt. tandem vibrating rollers.

Soil-cement.—Sub-base, fine sand with 8 per cent. cement; compacted by light vibrating rollers.

Granular base (concrete sections only).—1½-in. gravel with low clay content; compacted with 8-ton smooth-wheeled roller and sealed with a surface dressing of hot tar.

SURFACES.

Concrete.—It was estimated that 9 in. of reinforced concrete of normal strength would be required on a base 3 in. thick. The experimental section comprises thirty-five slabs, each 120 ft. long. Twenty-eight of these were made with a gravel aggregate and the concrete had minimum strengths at 28 days of 4000 lb. per square inch in compression and 450 lb. per square inch in bending; the remaining seven slabs were of concrete with minimum strengths at 28 days of 6500 lb. per square inch in compression and 650 lb. per square inch in bending.

The slabs were laid by spreading and finishing machines travelling on rails attached to side forms, 26 ft. apart, on concrete foundations. A longitudinal joint was sawn in the hardened concrete. Expansion joints were formed at the ends of the slabs, which were separated from each other by transition bays 15 ft. long. The sealing grooves over the expansion joints, and dummy joints at intervals of 15 ft. in the unreinforced concrete, were also sawn in the hardened concrete. All expansion joints were dowelled and tie-bars were used along the longitudinal joint, those in the reinforced slabs being incorporated in the reinforcement.

Two-course rolled asphalt (1½-in. wearing

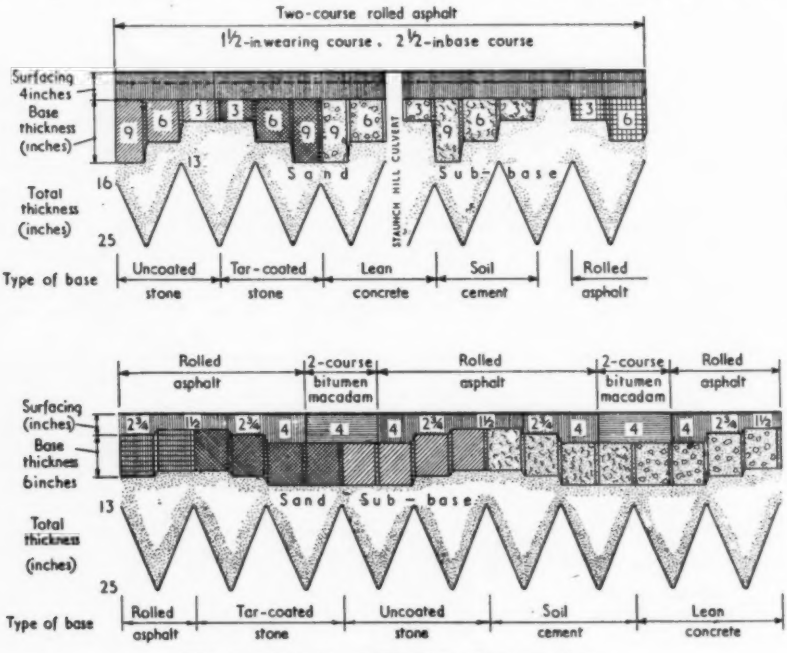


Fig. 2.—Details of Flexible Construction.

course on 2½-in. base).—Base course as rolled-asphalt base. Wearing course to B.S. 594, Table 5, Schedule 1; stone content 35 per cent., 1½-in. maximum size; asphaltic cement to Table 1, column 3.

Two-course rolled asphalt (1-in. wearing course on 1¾-in. base course).—Base course as before but stone ¾-in. maximum size. Wearing course as before.

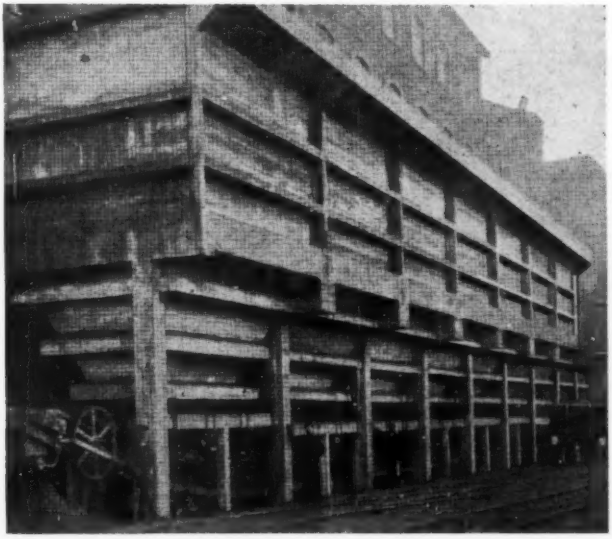
Single-course rolled asphalt.—As wearing course previously described. Coated chippings applied to all asphalt surfacings at the rate of 1 ton to 120-160 sq. yd.

Bitumen macadam (1½-in. wearing course on 2½-in. base).—Base to B.S. 1621, Table 3, stone 1½-in. nominal size; bitumen 300 penetration + 1 per cent. tar acids; 1 per cent. hydrated lime added to the aggregates before coating. Base covered with coated grit and surface-dressed with tar and ¼-in. crushed-rock chippings. Wearing course 1½-in. to B.S. 1621, Table 4; stone ¾-in. nominal size; binder as for base course; 2 per cent. hydrated lime added to the aggregate before coating.

FIFTY YEARS AGO.

From "CONCRETE AND CONSTRUCTIONAL ENGINEERING", October, 1909.

Coal-washing Bunkers, South Wales.



THE seven coal-washing bunkers illustrated have been erected at Bargoed, South Wales, for the Powell Duffryn Steam Colliery Co. on the Coignet system of reinforced concrete. The total capacity of the bunkers is 27,000 cu. ft. Their total length is 111 ft., width 14 ft., and height 31 ft. The work was secured in competition with steel bunkers. The contractors were Messrs. Heenan & Froude, Ltd., who are licensees of Messrs. Coignet, Ltd.

[We are informed by the National Coal Board, South-western Division, that these bunkers are still in use in 1959 and have given excellent service, and that very little repair work has been necessary.—ED.]

A Vibrating Roller.

A VIBRATING roller, now available in two sizes, is shown in *Fig.* 1. The smaller size is designed to be suitable for the compaction of bituminous materials, base concrete, sand, gravel, and chippings, and particularly for the reinstatement of trenches. The larger size is designed for the compaction of roads and runways. The advantages of the machines are stated to include the attainment of a high degree of compaction, simplicity of control and operation, and the ability to work on gradients up to 1 in 3.

The smaller machine is propelled by a 2¼-h.p. petrol engine and the larger by a 5-h.p. Diesel engine. The rollers vibrate alternately, the frequency of each being 3600 vibrations per minute, and the operating speed is one mile per hour. The machines are known as "Ace-Bomag" rollers, and are supplied by A.C.E. Machinery, Ltd.

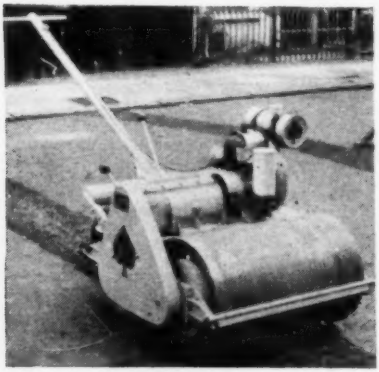


Fig. 1.

Large Concrete Pipes.

A PIPELINE more than six miles long is being laid at depths up to 200 ft. below the Pacific Ocean off the coast of California, U.S.A. Some 5½ miles will comprise precast reinforced concrete pipes 24 ft. long and generally 14 ft. in diameter and weighing 85 tons; the remainder will be in lengths of 12 ft. or 14 ft. The walls of the pipes are 10 in., 12 in., or 15 in. thick, depending upon their position in the pipeline. Rubber gaskets are used at the joints. The pipes are being made by the United Concrete Pipe Corporation.

Polythene Covers for Curing Concrete.

THE improvement in the conditions of dampness during the curing of concrete covered by sheets of plastic is illustrated by observations made recently by Mr. Arthur Martin. Some 2-in. prestressed concrete planks were manufactured by the long-line method, and after casting were covered with polythene sheets until the tension in the wires was relaxed seven days later. During the morning after the planks were cast, globules and patches of water had collected on the underside of the sheets and on the top of the concrete. This moisture remained visible throughout the period of seven days, so that the slabs were constantly under damp conditions, the polythene sheets forming an impermeable cover that prevented evaporation of the water. A Belgian sulphate-resistant cement was used for the planks. Test cubes cured under the same conditions had strengths of 2600 lb. per square inch at three days, 4000 lb. at five days, 5900 lb. at seven days and 10,900 lb. per square inch at 28 days.

Lectures on Building.

THE following lectures have been arranged by the Ministry of Works. Admission is free.

Work Study in the Building Industry. By R. Geary. College of Further Education, Greenclose Lane, Loughborough. October 15. 7.15 p.m. By R. G. Lacey. 22 George Street, Edinburgh. October 21. 7.15 p.m.; and at Royal Hotel, Union Street, Dundee. October 22. 7.15 p.m.

R.I.B.A. Form of Contract. By I. N. D. Wallace. Technical College, Doncaster. October 21. 7.15 p.m.; and at College of Technology, Wulfruna Street, Wolverhampton. October 27. 7.15 p.m.

New Motorway in Staffordshire. College of Building, Cauldon Place, Stoke-on-Trent. October 20. 7.15 p.m.

Thermal Insulation of Buildings. By J. Lawrie. Technical College, Southampton. October 22. 7.15 p.m.

Introduction to Site Costing for Builders. By A. E. Chittenden. Medway College of Technology, Maidstone Road, Chatham. October 23. 7.15 p.m.

Settlement of Buildings. By S. R. Rosenak. Willesden Technical College, London, N.W.10. October 27. 7.15 p.m.